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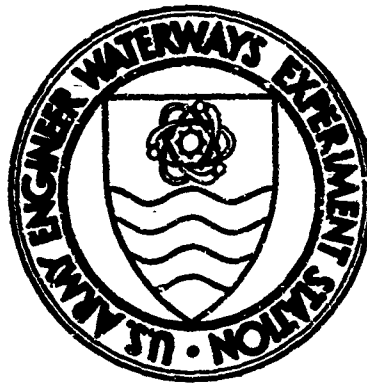
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TECHNICAL REPORT NO. 3-652

# MEASURING SOIL PROPERTIES IN VEHICLE MOBILITY RESEARCH

Report 4

## RELATIVE DENSITY AND CONE PENETRATION RESISTANCE

by

K.-J. Melzer



July 1971

Sponsored by U. S. Army Materiel Command

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## FOREWORD

The study reported herein was conducted in 1969 as part of Department of the Army Project 1T061102B52A, "Research in Military Aspects of Terrestrial Sciences," Task 01, "Military Aspects of Off-Road Mobility," under the sponsorship and guidance of the Research, Development and Engineering Directorate, U. S. Army Materiel Command.

The study was conceived and directed by Dr. K.-J. Melzer and was performed by personnel of the Mobility Research Branch, Mobility and Environmental Division, U. S. Army Engineer Waterways Experiment Station (WES). The work was conducted under the general supervision of Messrs. W. G. Shockley and S. J. Knight, and under the direct supervision of Drs. D. R. Freitag and K. W. Wiendieck. Dr. Melzer prepared this report.

COL Levi A. Brown, CE, and COL Ernest D. Feixotto, CE, were Directors of the WES during this study and preparation of this report. Messrs. J. B. Tiffany and F. R. Brown were Technical Directors.

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# NOTATION

$a_1, a_2$	Constants in equation 12
$A_c, A_s$	Base and skin areas, respectively, of cone or pile, $\text{cm}^2$
$b$	Width of cone base, cm
$c_1, c_2$	Test constants (equation 4), dimensionless
$C_u$	Coefficient of uniformity of the soil, dimensionless: $d_{60}/d_{10}$
$d$	Diameter of base of cone, cm
$d_m$	Mean diameter of soil grains, mm
$d_q$	Depth factor, dimensionless
$d_{10}, d_{50}, d_{60}$	Soil grain diameter at 10, 50, and 60 percent finer by weight, mm
$D$	Total penetration depth, cm
$D^*$	Compactibility of the soil, percent: $\frac{e_{\max} - e_{\min}}{e_{\min}} \times 100$
$D_c$	Critical depth of penetration, cm
$D_{cI}, D_{cII}$	Critical depth of penetration according to methods I and II, respectively (fig. 3), cm
$D_r$	Relative density of the soil, percent
$D_l$	Registered penetration depth, cm
$e$	Natural void ratio in the soil: $\frac{\gamma_d \gamma_s}{\gamma_a} - 1$
$e_{\max}, e_{\min}$	Void ratio in the soil in its loosest and densest states, respectively
$G$	Cone penetration resistance gradient, $\text{MN}/\text{m}^3$
$GW$	Gravel, classified according to the Unified Soil Classification System
$n$	Number of data points in a relation
$n_q$	Bearing capacity factor for depth, dimensionless

$N'_q$	Bearing capacity factor including depth factor = $N_q d_q$ , dimensionless
$q_c$	Penetration resistance of a cone or the base of a pile, $\text{kN/m}^2$
$\bar{q}_c$	Average 0- to 15-cm cone penetration resistance, $\text{kN/m}^2$
$q_s$	Specific skin friction, $\text{kN/m}^2$
$Q_c, Q_s, Q_{\max}$	Base, skin, and ultimate loads, respectively, N
$r$	Correlation coefficient, dimensionless
$s_q$	Shape factor, dimensionless
$s_{y \cdot x}$	Standard deviation
SP-SM, SP	Fine sands, classified according to Unified Soil Classification System
$w$	Moisture content, percent
$x, y$	Variables
$\bar{x}, \bar{y}$	Mean values of the variables $x$ and $y$
$\beta$	Inclination angle in the $D_r$ versus $\bar{q}_c$ relation
$\gamma$	Unit weight of soil, $\text{kN/m}^3$
$\gamma_d$	Dry unit weight of soil, $\text{kN/m}^3$
$G_s$	Specific gravity, dimensionless
$\gamma_w$	Unit weight of water, $\text{kN/m}^3$
$\alpha$	Apex angle, deg
$\phi$	Angle of internal friction, deg

## CONVERSION FACTORS, METRIC TO BRITISH UNITS OF MEASUREMENT

Metric units of measurement used in this report can be converted to British units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
centimeters	0.3937	inches
meters	3.2808	feet
square centimeters	0.1550	square inches
newtons	0.2248	pounds (force)
kilonewtons per square meter	0.1450	pounds per square inch
kilonewtons per cubic meter	6.3659	pounds per cubic foot
millimeters	0.0394	inches
meganeutons per cubic meter	3.684	pounds per cubic inch

## SUMMARY

Relations between cone penetration resistance and relative density were developed by means of statistical analysis (correlation calculation) for three cohesionless soils: Yuma sand, mortar sand, and Bayou Pierre sand. These relations were evaluated from direct measurements of relative density and results of tests with the U. S. Army Engineer Waterways Experiment Station (WES) standard cone penetrometer. Most of the data for Yuma and mortar sands had already been obtained as part of the soil-tire performance tests previously conducted at the WES. However, several special laboratory tests in molds with both sands were conducted to control and extend the existing range of data. The results in Bayou Pierre sand were obtained from laboratory tests conducted especially for this study.

The relations established between relative density and cone penetration resistance and its gradient, respectively, averaged over the 0- to 15-cm depth, depend on the grain size and compactibility of the soil. The cone penetration resistance increases with increasing soil mean grain diameter and decreasing compactibility. The critical depth of penetration affects the results within the considered depth range only in loose and very loose sands. A qualitative theoretical explanation of what occurs during the penetration of a cone into a cohesionless medium is given.

## MEASURING SOIL PROPERTIES IN VEHICLE MOBILITY RESEARCH

### RELATIVE DENSITY AND CONE PENETRATION RESISTANCE

#### PART I: INTRODUCTION

##### Background

1. One of the main deficiencies in off-road mobility research is the lack of a set of valid relations between vehicle performance and soil properties. These relations require a proper method for adequately measuring pertinent soil characteristics; therefore, in the last 20 years, many mobility research organizations have tried to develop devices for measuring certain soil properties in situ. For instance, the Military Vehicles and Engineering Establishment in Great Britain uses a vane shear apparatus to measure soil strength;<sup>1</sup> and the U. S. Army Tank-Automotive Command uses the bevameter, with which plate penetration tests and horizontal shear tests are performed.<sup>2</sup> From results obtained with these devices, soil parameters are derived for use in approaches to the problem of soil-vehicle interaction.

2. In 1945 the U. S. Army Engineer Waterways Experiment Station (WES) introduced the cone penetrometer, adapted from the Proctor needle, into mobility research.<sup>3</sup> Contrary to bevameter readings, the cone penetrometer readings (resistance values) were not considered basic soil properties to be used in a semitheoretical approach, but a convenient measure of soil strength (cone index).

3. Cone index was defined as average penetration resistance over a depth of 0 to 15 cm\* in both cohesive and cohesionless soils.<sup>4,5</sup> Later, it came to be used to represent the strength of cohesive soils only; for cohesionless soils, the cone index gradient was introduced,<sup>4</sup> i.e. the rate of penetration resistance increase averaged over a depth of 15 cm. (As a result of using metric units, the terms cone penetration resistance and cone

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\* A table of factors for converting metric units of measurement to British units is given on page xi.

penetration resistance gradient have replaced cone index and cone index gradient.)

4. Because many organizations dealing with off-road mobility research use their own systems for measuring soil properties, comparison and interpretation of the performance of terrain-vehicle systems based on the various approaches are difficult. For that reason, a committee for off-road ground mobility research appointed by the Chief of Research and Development, U. S. Army, in 1959 recommended that the WES conduct studies to correlate the various measurements derived by the different approaches for cohesive and cohesionless soils.<sup>1,6</sup>

5. Developments in soil mechanics in the last 30 years have shown that tests with dynamic as well as static penetrometers are proper techniques for quick determination of the subsoil properties that influence the bearing capacity and settlement of foundations. Therefore, one aim of research has been to establish relations for cohesionless soils between their relative density\* and the results from the various penetrometer tests, primarily the cone penetration resistance of the soils. This relation was selected because (a) bearing capacity and settlement can at least be estimated if the order of magnitude of the relative density is known, and (b) determination of the in situ unit weight or relative density by direct measurement is often difficult in subsoil exploration.

6. The first relations established between relative density and cone penetration resistance seemed to be valid for all sands.<sup>7</sup> Later, cone penetration resistance was found to depend not only on the relative density, but also on the type of cohesionless soil being penetrated.<sup>8-11</sup> Accordingly, attempts were made to determine the soil properties that affect

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\* For many problems, dry unit weight, or void ratio, alone is not sufficient to characterize the density of cohesionless soils; the possible density range has to be taken into account. Relative density  $D_r$  is, in many respects, a more correct description of the actual properties of cohesionless soils, since its definition,<sup>6</sup>  $D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}}$ , includes not only void ratio  $e$  in the natural state, but also the possible void ratio  $e_{\max}$  for the loosest state and the void ratio  $e_{\min}$  for the densest state of the material.

the penetration resistance-relative density relation. Mean grain size and compactibility were found to have considerable influence.<sup>12,13</sup> A complete review of the recent research in this field is given in references 12, 14, and 15.

#### Purpose

7. The purpose of the study reported herein was to:
  - a. Qualitatively describe by theoretical means the behavior of cohesionless soil when it is penetrated by a cone.
  - b. Establish a relation between relative density and cone penetration resistance (gradient) for sands, with mean grain size and compactibility taken into account.

#### Scope

8. Three sands were investigated in the laboratory: (a) a desert sand from near Yuma, Arizona (Yuma sand), (b) a washed sand from an alluvial plain (mortar sand), and (c) a river-deposited sand (Bayou Pierre sand). All three sands were tested in the air-dry state. Most of the data for the Yuma and mortar sands had been collected in soil-tire performance tests conducted at the WES in very carefully prepared soil bins.<sup>16</sup> To complete the data collection, penetration tests in Bayou Pierre sand and a few control tests in Yuma and mortar sands were conducted in cylindrical steel molds 38 cm in inside diameter and 30 cm high. The sands were placed in the molds such that the consistency in each mold was as uniform as possible. Relative densities ranged from very loose to very dense. The WES standard cone penetrometer was used in all the tests, and average cone penetration resistance and gradients were determined from the 0- to 15-cm depth. Penetration speed was 0.03 m/sec.

## PART II: THEORETICAL CONSIDERATIONS

### General Bearing Capacity Formula

9. The basic bearing capacity formula for a pile, or a penetrometer considered to be a model pile, can be stated generally as

$$Q_{\max} = Q_c + Q_s = q_c A_c + q_s A_s \quad (1)$$

where

$Q_{\max}$  = ultimate load

$Q_c$  = base load

$Q_s$  = skin load

$q_c$  = penetration resistance of a cone or the base of a pile

$A_c, A_s$  = area of the base and the skin, respectively

$q_s$  = specific skin friction

For the present investigations, only the cone penetration resistance is important, since the skin friction can be neglected for small depths of penetration.<sup>8,12</sup> Therefore, for the cone penetration tests in sands, equation 1 can be written:

$$Q_{\max} = Q_c = q_c A_c \quad (2)$$

10. There are many different solutions to the problem of evaluating cone penetration resistance theoretically, and nearly all of them are based on the classical theory of plasticity.<sup>17,18</sup> The main assumptions of these solutions are:

- a. The volume change in the soil caused by shearing is negligible, i.e. the soil is incompressible; this assumption is approximately valid for dense, cohesionless soils only.
- b. There are only plastic deformations.
- c. The angle of internal friction along the rupture surfaces is independent of the stress acting normal to the rupture lines.

11. Theoretical, calculated bearing capacity of deep foundations has

seldom agreed with in situ, measured bearing capacity. Efforts have been made to find sophisticated solutions to this problem,<sup>19-21</sup> but a generally valid one has not been found. One solution was found applicable in some cohesionless soils, but failed in others.<sup>22</sup>

12. Although there is no generally valid theoretical solution that yields reasonable quantitative results (and the difficulties in reaching this goal seem to increase with time rather than decrease),<sup>23</sup> a relation between cone penetration resistance and relative density can be justified by existing solutions based on the theory of plasticity. Furthermore, at least a qualitative explanation of what occurs during the penetration of a cone into cohesionless soils should be possible.

13. If soil cohesion is zero and the width/depth ( $b/D$ ) ratio of a cone is negligibly small, the theoretical bearing capacity equation for the cone can be stated:

$$q_c = \frac{Q_c}{A_c} = \gamma D N_q d_q s_q = \gamma D N_q^* s_q \quad (3)$$

where

$\gamma$  = unit weight of the soil

$n$  = depth of penetration

$N_q$  = bearing capacity factor for depth

$d_q$  = depth factor

$s_q$  = shape factor, usual  $\approx 1.5$  for a circular base

$N_q^* = N_q d_q = f(\phi)$  for constant depth

If the penetration depth and the diameter of the penetrometer are held constant, cone penetration resistance  $q_c$  in equation 3 depends on  $N_q^*$  only. According to plasticity theory, the bearing capacity factor  $N_q^*$  is related to the angle of internal friction  $\phi$ . Thus, a relation between cone penetration resistance and angle of internal friction exists (fig. 1).

#### Relative Density and Friction Angle

14. Various authors have demonstrated, by theoretical and empirical means, that the angle of internal friction  $\phi$  depends on the void ratio

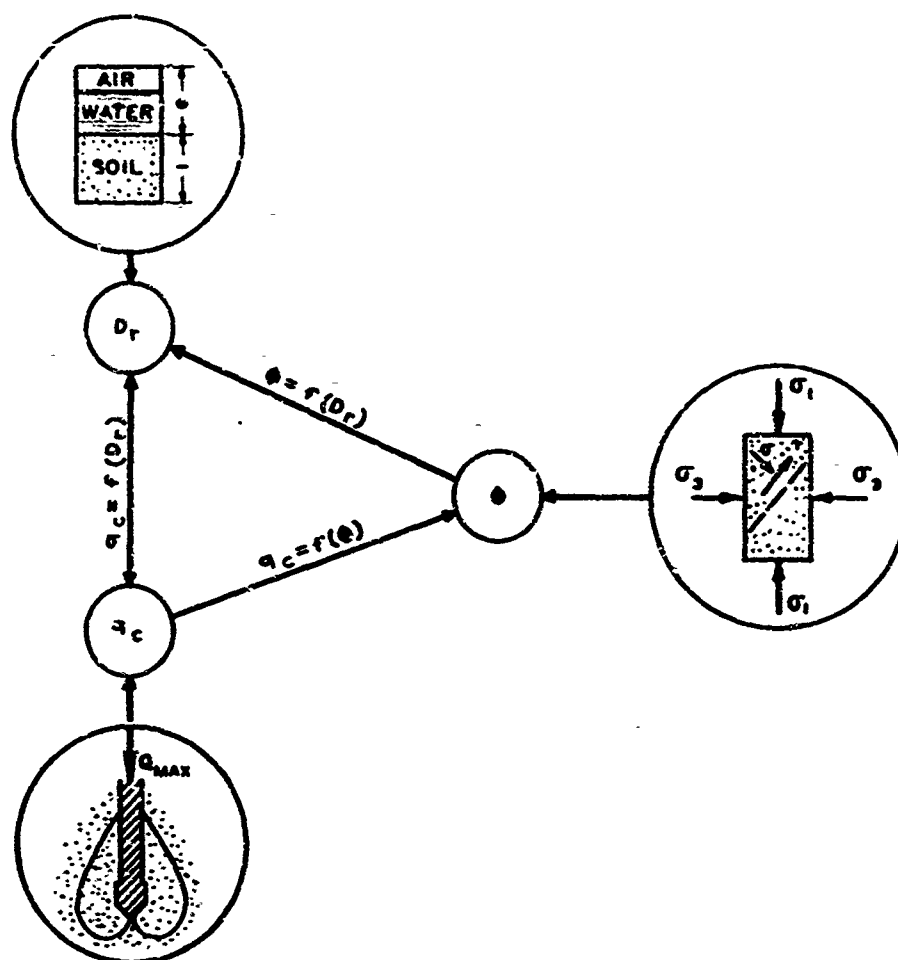


Fig. 1. Relation between cone penetration resistance  $q_c$  and relative density  $D_r$

or the relative density (fig. 1), even if a generally valid solution has not yet been found.<sup>24,25</sup> One proposed equation is<sup>24</sup>

$$\cot \phi = c_1 e + c_2 \quad (4)$$

where

$c_1, c_2$  = constants obtained by testing  
 $e$  = initial void ratio

#### Relative Density and Cone Penetration Resistance

15. The bearing capacity factor  $N_q^*$  in equation 3 can be replaced

by a function of relative density, so that a relation between cone penetration resistance and relative density is obtained (fig. 1):

$$q_c = \gamma D f(D_r) s_q \quad (5)$$

This qualitative derivation shows<sup>12</sup> that relating relative density and cone penetration resistance by empirical means is feasible.

### Critical Depth

16. Critical depth  $D_c$  is the depth at which the penetration resistance of the base of a pile, or the penetration resistance of a cone, ceases to increase rapidly (fig. 2d). Below that depth, the resistance increases only slightly, and the rate of increase remains constant. This phenomenon has been observed often in tests with piles and penetrometers<sup>12,20,26</sup> and may be important in judging soil density profiles with the WES cone penetrometer.

17. According to the theoretical considerations of De Beer,<sup>12,20</sup> the results obtained during a penetration can be explained qualitatively as follows.\* Penetrating a cohesionless soil, a penetrometer produces various states of failure, as shown by the rupture lines in figs. 2a-2c. At the beginning of the test, the rupture pattern for very shallow penetration depths corresponds to that of a shallow foundation (fig. 2a), which means that  $d_q = 1.0$  (equation 3) and

$$N_q^* = N_q = \tan^2 \left( 45^\circ + \frac{\phi}{2} \right) e^{\pi \tan \phi} \quad (6)$$

18. With increasing depth, the shearing resistance along the rupture surfaces above the penetrated depth (fig. 2b) cannot be neglected. For each depth smaller than the critical depth, i.e. where the rupture lines cut the penetrometer shaft,<sup>12,20</sup>

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\* This specific explanation has been chosen because it leads to reasonable qualitative results. This does not mean that there are not other possible solutions.

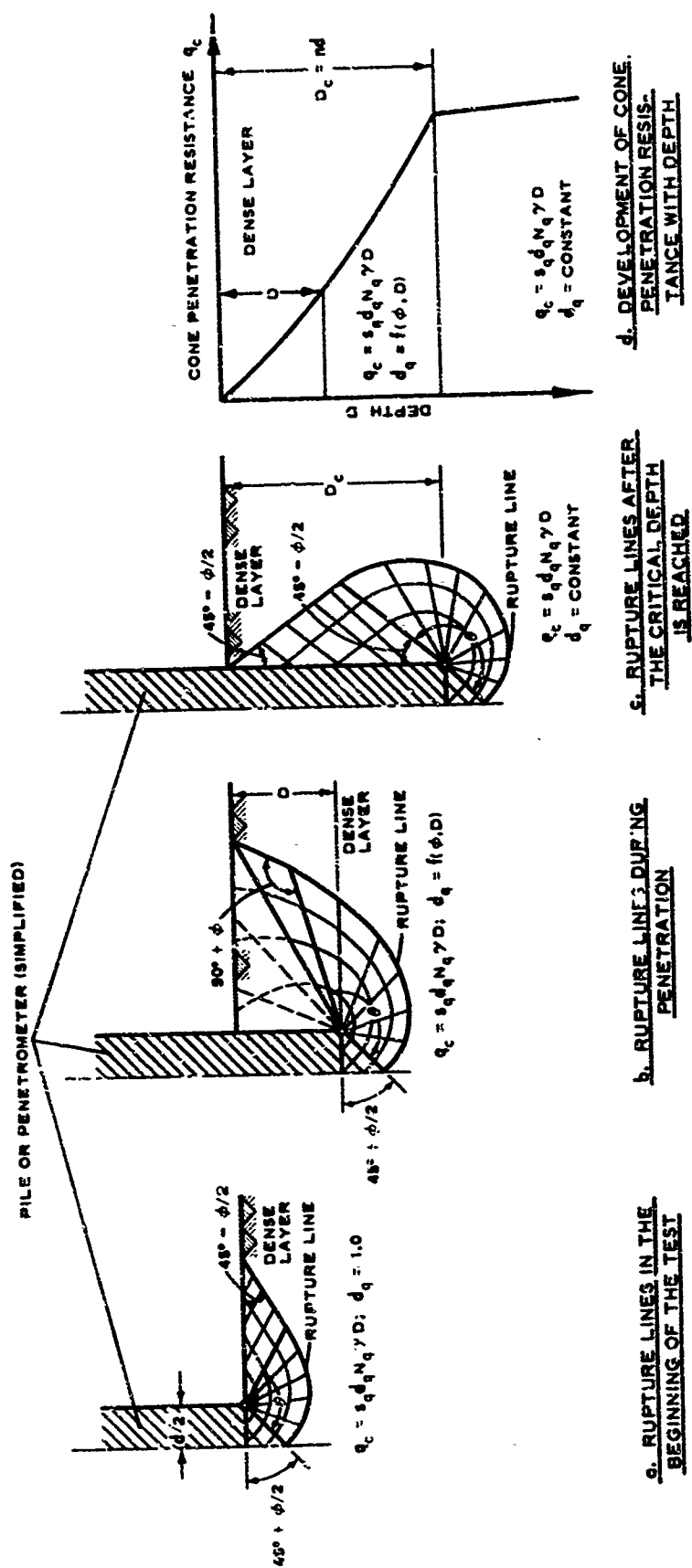


Fig. 2. Development of the rupture lines and of cone penetration resistance with depth (simplified from reference 20)

$$N'_q = \tan^2 \left( 45^\circ + \frac{\phi}{2} \right) e^{2\theta \tan \phi} \quad (7)$$

with  $\pi/2 \leq \theta < \pi$  (figs. 2a and 2b),  $\theta$  being the apex angle. The total bearing capacity factor  $N'_q$  can be separated into the bearing capacity factor  $N_q$  for a shallow foundation and the depth factor  $d_q$  as follows:<sup>12</sup>

$$N'_q = \frac{\tan^2 \left( 45^\circ + \frac{\phi}{2} \right) e^{2\theta \tan \phi}}{\tan^2 \left( 45^\circ + \frac{\phi}{2} \right) e^{\pi \tan \phi}} \cdot \tan^2 \left( 45^\circ + \frac{\phi}{2} \right) e^{\pi \tan \phi}$$

$$N'_q = e^{(2\theta - \pi) \tan \phi} \cdot \tan^2 \left( 45^\circ + \frac{\phi}{2} \right) e^{\pi \tan \phi} \quad (8)$$

where

$$e^{(2\theta - \pi) \tan \phi} = d_q \quad (9)$$

and

$$\tan^2 \left( 45^\circ + \frac{\phi}{2} \right) e^{\pi \tan \phi} = N_q \quad (\text{equation 6})$$

The apex angle  $\theta$  depends on the depth  $D$ ; thus  $d_q$  does not increase linearly with depth (fig. 2d), nor does the cone penetration resistance  $q_c$  (equation 3). The critical depth/diameter ratio  $D_c/d$  is given by Jaky<sup>27</sup> as:

$$\frac{D_c}{d} = \tan \left( 45^\circ + \frac{\phi}{2} \right) e^{\pi \tan \phi} \quad (10)$$

19. Below the critical depth, the depth factor remains constant

$$d_{qc} = e^{\pi \tan \phi} \quad (11)$$

so the penetration resistance increases only because of the increasing overburden pressure  $\gamma D$ , according to equation 3, and the increase is

therefore much smaller than that above the critical depth.\*

20. The critical depth increases with increasing angle of internal friction (relative density). From equation 10,  $D_c/d$  ratios of 2, 10, and 30 were obtained for  $\phi = 10$  deg, 30 deg, and 40 deg, respectively. The order of magnitude of these values was verified by large-scale testing, at least for dense and very dense cohesionless soils.<sup>12,20</sup> It must be pointed

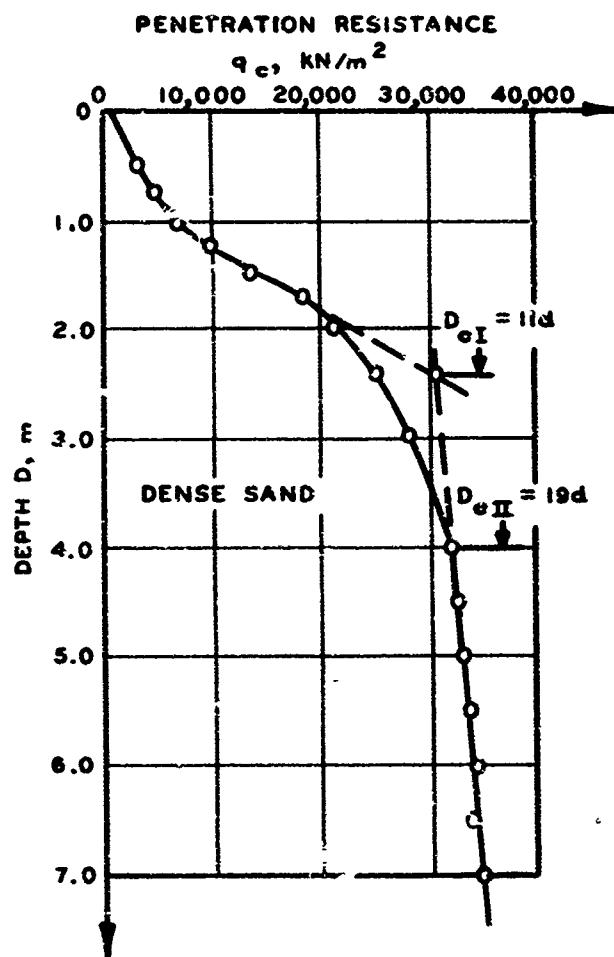


Fig. 2 Variation of penetration resistance with depth for a pile ( $d = 21.6$  cm) (reference 26) and determination of the critical depth according to methods I and II

out again that the explanation of the critical depth is based on the theory of plasticity; therefore, it is valid only for incompressible materials to which dense soils approximately correspond.

21. When the theoretical increase of the cone penetration resistance with depth (fig. 2d) is compared with the increase measured in actual testing (fig. 3), an approximate agreement in the general trend of the curves results; however, the critical depth cannot be clearly recognized in the measured test results. Besides the plastic deformations (paragraph 10), which are dominant in the case of dense sands, deformations caused by compression come into the picture for looser sands.

22. Two methods for

\* In some instances of actual testing, this small increase in the penetration resistance below the critical depth was not observed.<sup>28</sup> This was explained by arching of the sand above the penetrometer base.

evaluating critical depth are possible (fig. 3):

- a. Method I. The point of intersection of the linearly extrapolated upper and lower parts of the curve yields the critical depth  $D_{cI}$  on the depth axis. This results in a curve that corresponds to the expected theoretical curve of cone penetration resistance versus depth.
- b. Method II. The critical depth is reached at that depth below which the rate of increase of the cone penetration resistance with depth remains constant ( $D_{cII}$ ). This corresponds to the general definition of the critical depth in paragraph 16.

### PART III: TESTS

#### Sands

23. Yuma and mortar sands are uniformly graded, fine sands classified SP-SM and SP, respectively, according to the Unified Soil Classification System. Bayou Pierre sand is somewhat less uniform than these and is classified SP. Gradation and soil property data of the three sands are given in fig. 4. There was generally a good agreement of  $e_{\max}$  and  $e_{\min}$  values for these sands when the void ratios in the loosest and the densest states were compared with statistical evaluations of the limiting void ratios of sands.<sup>12,24</sup>

#### Test Equipment

24. Steel cylindrical molds were used to hold the test materials. The outside diameter of each mold was 39 cm, the inside diameter 38 cm, and the height 30 cm.

25. The WES standard cone penetrometer was used in all tests. The cone had a 30-deg apex angle and a base diameter of 2.03 cm (base area  $3.26 \text{ cm}^2$ ), and was mounted on a shaft with a smaller diameter to reduce skin friction. A 0.95-cm-diam shaft was used in the Yuma sand tests, and a 1.59-cm-diam shaft was used in the mortar and Bayou Pierre sand tests. Shafts with different diameters were used because, in the Yuma sand tests, an electrical event marker activated the pressure-measuring and depth-measuring devices, and in the mortar and Bayou Pierre sand tests, a photo-cell mounted in the base of the cone made a shaft with a large diameter necessary. Nevertheless, results obtained from control tests with both shafts in Yuma sand were compared and showed no influence of the shaft diameter.

26. The penetrometer was pushed into the soil by a hydraulic jack (fig. 5) at a speed of 0.03 m/sec (standard speed). The penetration resistance was measured continuously through the 0- to 15-cm depth by a load cell mounted at the top of the penetrometer shaft and was registered on an



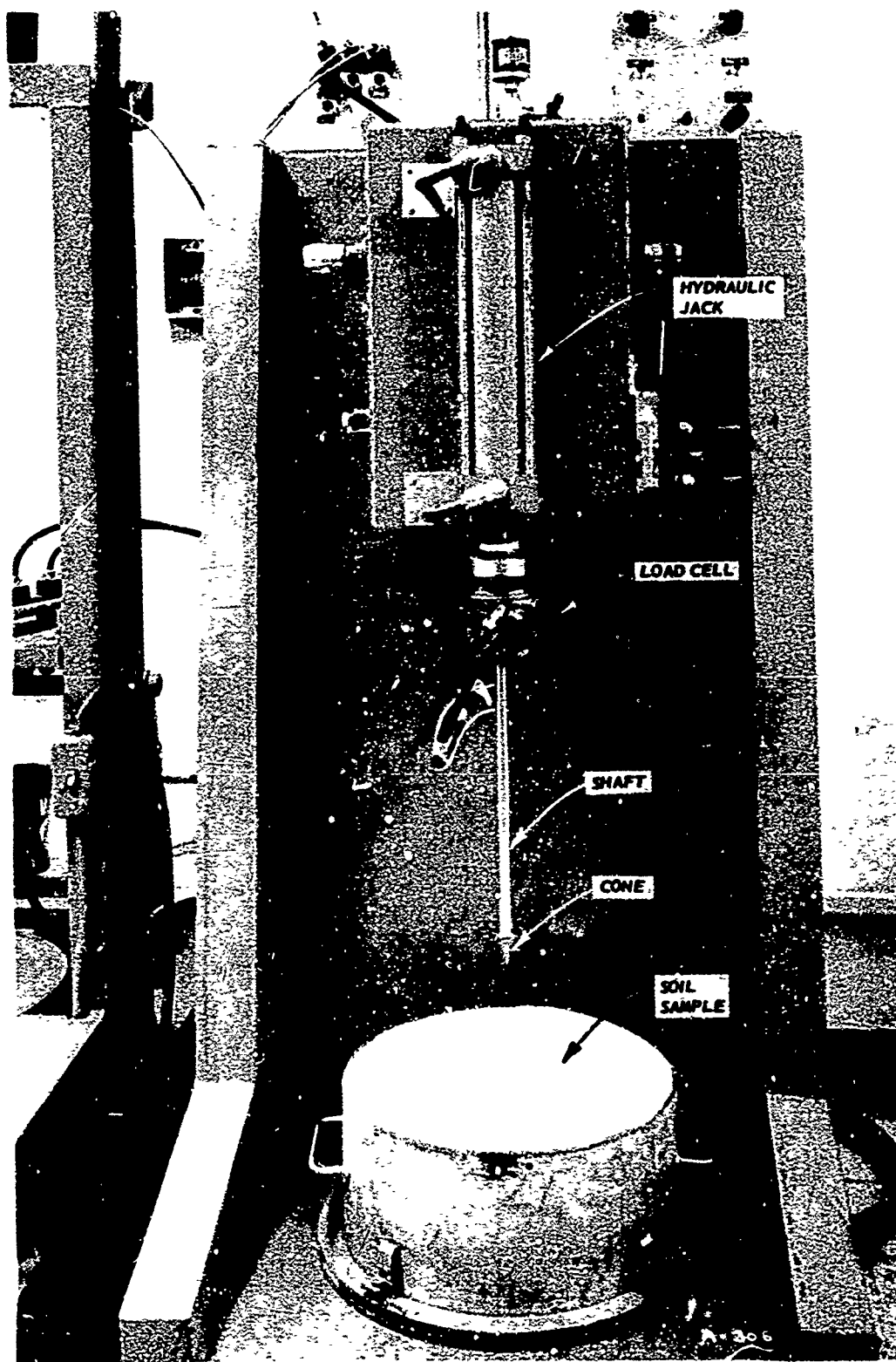


Fig. 5. Laboratory penetration test setup

x-y recorder. Because of the two different methods of activating the registration devices (paragraph 25), the cone penetration resistance versus depth diagrams were recorded for Yuma sand from that point where the tip of the cone touched the soil surface, and for mortar and Bayou Pierre sands from that point where the base of the cone was flush with the soil surface. (The latter is the normal procedure for registering depth with the WES standard cone penetrometer; thus, the 0- to 15-cm average cone penetration resistance is actually a resistance averaged over a depth 4 to 19 cm below the soil surface.)

#### Test Procedures

27 Two methods were used to fill the test molds as uniformly as possible to achieve the desired relative densities. For loose and medium-dense samples, the sand was placed with a funnel or through a sieve that covered the area of the mold, and the height of fall was varied. For denser samples also, the molds were filled through a sieve, but they were compacted by tapping the outside of the molds with a hammer.

28. After each mold was prepared, unit weight  $\gamma$  and moisture content  $w$  were determined, the corresponding void ratio  $e$  and relative density  $D_r$  were calculated, and the cone penetration test was conducted.

## PART IV: ANALYSIS OF TEST RESULTS

### Results Obtained

29. Representative curves of cone penetration resistance  $q_c$  versus depth  $D$  (0 to 15 cm) from tests in Yuma, mortar, and Bayou Pierre sands are given in plates 1a, 1b, and 1c, respectively.

30. Average cone penetration resistance  $\bar{q}_c$  and average gradient  $G$  values were determined and are listed for all tests in tables 1-3, together with corresponding values of dry unit weight  $\gamma_d$ , void ratio  $e$ , and relative density  $D_r$ . The ranges of these variables are given in the following tabulation:

Sand	Dry Unit Weight $\gamma_d$ , kN/m <sup>3</sup>		Void Ratio $e$		Relative Density $D_r$ , %		Average Cone Penetration Resistance $\bar{q}_c$ , kN/m <sup>2</sup>		Average Penetration Resistance Gradient $G$ , MN/m <sup>3</sup>	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Yuma	13.773	16.282	0.610	0.901	5.8	99.4	28	475	0.3	5.8
Mortar	13.881	16.422	0.595	0.887	6.3	93.2	35	455	0.4	6.3
Bayou Pierre	15.676	18.423	0.419	0.658	0.0	97.3	53	698	0.6	9.1

### Analysis of Data

#### Critical depth

31. The critical depth values in this study (plate 2) were evaluated from the cone penetration resistance versus depth curves, according to the two methods given in paragraph 22 and fig. 3. The critical depth was measured from the soil surface, and could be defined only for tests with relative densities less than roughly 35 percent. At greater densities, critical depth either was not well defined, or was not reached because of the limited height of the molds (30 cm).

32. The critical depth was divided by the diameter of the cone to obtain the ratio  $D_c/d$  for  $D_c$  derived by both methods. These ratios were plotted versus relative density for all sands tested, and the

relations separated according to method I (plate 3a) and method II (plate 3b). Even though there was some scattering of the data, the relation indicates that the  $D_c/d$  ratio tends to increase with increasing relative density.

#### Void ratio

33. The purpose of this study was not to evaluate the relation between void ratio and cone penetration resistance, but between relative density and cone penetration resistance. Nevertheless for completeness, void ratio and dry unit weight were plotted versus average cone penetration resistance (plate 4) and penetration resistance gradient (plate 5) for the three sands tested. All the plots show the expected relations. Cone penetration resistance and penetration resistance gradient increased with decreasing void ratio. Results obtained from other penetration tests conducted in mortar sand<sup>29</sup> agree with these results (plate 5b).

#### Relative density

34. To establish quantitative relations between relative density and average cone penetration resistance and gradient, respectively, the statistical method of correlation calculation<sup>30-32</sup> was used. After trials with various types of equations, it was found that the following general form fits the data best for a specific sand type:

$$y = a_1 \log x + a_2 \quad (12)$$

where

$y$  = relative density  $D_r$ , %

$a_1$  and  $a_2$  = constants

$x$  = average cone penetration resistance  $\bar{q}_c$ ,  $\text{KN/m}^2$ , or  
cone penetration resistance gradient  $G$ ,  $\text{KN/m}^2/\text{in.}$

35. The constants  $a_1$  and  $a_2$  for the three sands are listed in the following tabulation, together with the corresponding mean values, standard deviations, and correlation coefficients:

Type of Sand	No. of Tests	Variables*		Constants		Mean Values		Standard Deviation $\pm s_{y \cdot x}$	Correlation Coefficient $r$
		$y$	$x$	$a_1$	$a_2$	$\bar{y}$	$\bar{x}$		
Yuma	91	$D_r$	$\bar{q}_c$	71.2	-88.6	67.6	1.183	6.3	0.960
		$D_r$	G	71.1	+51.6	67.6	0.225	6.8	0.952
Mortar	37	$D_r$	$\bar{q}_c$	75.5	-106.0	63.5	1.245	9.0	0.941
		$D_r$	G	75.0	+39.3	63.5	0.322	9.2	0.938
Bayou Pierre	36	$D_r$	$\bar{q}_c$	77.2	-119.2	55.8	1.267	4.4	0.988
		$D_r$	G	77.0	+29.5	55.8	0.342	4.2	0.989

\* Dimensions:  $D_r$  in %;  $\bar{q}_c$  in  $\text{kN/m}^2$ ; G in  $\text{MN/m}^3$ .

Substitution of values from the tabulation above for  $y$ ,  $x$ ,  $a_1$ , and  $a_2$  in equation 12 yields the following equations.

a. Yuma sand.

$$D_r = 71.2 \log \bar{q}_c - 88.6 \quad (13a)$$

$$D_r = 71.1 \log G + 51.6 \quad (13b)$$

b. Mortar sand.

$$D_r = 75.5 \log \bar{q}_c - 106.0 \quad (14a)$$

$$D_r = 75.0 \log G + 39.3 \quad (14b)$$

c. Bayou Pierre sand.

$$D_r = 77.2 \log \bar{q}_c - 119.2 \quad (15a)$$

$$D_r = 77.0 \log G + 29.5 \quad (15b)$$

Relations between  $D_r$  and  $\bar{q}_c$  and between  $D_r$  and G are plotted in plates 6 and 7, respectively.

36. The two correlation coefficients for the same sand are very close to each other, regardless of whether  $\bar{q}_c$  (equations 13a, 14a, 15a) or  $G$  (equations 13b, 14b, 15b) is used as variable  $x$ . Therefore, for the purpose of evaluating  $D_r$  it does not matter whether  $\bar{q}_c$  or  $G$  is calculated from the cone penetration resistance-depth diagram. Furthermore, constants  $a_1$  and  $a_2$  are different for the three sands, i.e. the same values of  $\bar{q}_c$  and  $G$  indicate different relative densities for each sand.

### Interpretation of Data

#### Critical depth

37. The curves in plate 2, which represent the cone penetration resistance-depth relations over the entire depth of the test molds, show results that were expected. The curves are first concave upward and then convex. The rate of increase of the cone penetration resistance with depth reaches its maximum within the convex portion. The rate of increase then decreases with depth, reaching its minimum at the point of the critical depth, and remains nearly constant below that depth. Thus, the curves are qualitatively similar to the theoretical cone penetration resistance-depth curve in fig. 2d\* and to those observed in earlier testing, e.g. with normal-size piles (fig. 3).

38. Furthermore, the critical depth increases with increasing relative density (plate 2). This tendency becomes clearer when the relations between relative density and the  $D_c/d$  ratios obtained from results with all three sands (plate 3) are considered. The  $D_r-D_c/d$  relations agree, at least qualitatively, with the theoretical relation (equation 10), wherein the critical depth does not increase linearly with increasing angle of internal friction, i.e. with increasing relative density (paragraphs 14 and 18).

39. For practical purposes, the registered depth range of 0 to 15 cm ( $D_c/d = 0$  to 9.5) over which the cone penetration resistance and the gradient were averaged lies above the critical depth for medium-dense to very dense sands, so for most cases of interest in mobility research,  $\bar{q}_c$  and

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\* At least concerning the concave portion and the portion below the critical depth.

G are reliable indicators of strength in the 0- to 15-cm range. The critical depth occurs within the 0- to 15-cm depth only in loose and very loose materials.

#### Relative density

40. Because a quantitative explanation of the different results for the three sands by theoretical means did not promise much success, a more qualitative reasoning was applied to explain these results.

41. When a cone penetrates a cohesionless soil, the grains are displaced. The displacement forces depend not only on the relative density, but also on the compactibility of the soil, in that the grains in a highly compactible sand can be displaced with less difficulty than those in a sand with low compactibility at the same relative density. This results in higher cone penetration resistance in soils with low compactibility. Furthermore, uniform soils (low compactibility) generally have higher friction angles than nonuniform soils (high compactibility),<sup>33</sup> which leads, according to bearing capacity theories, to the same effects mentioned above. Earlier investigations with penetrometers for deep penetrations<sup>12,13</sup> support this reasoning.

42. Also, cone penetration resistance will be greater in a soil with large-diameter grains than in a soil with smaller grains. For instance, when a cone penetrates a gravel and a sand, both of which have the same relative density and compactibility, cone penetration resistance will be much greater in the gravel.

43. The above-mentioned considerations agree with earlier investigations,<sup>12,13</sup> which showed that compactibility  $D^*$  and mean diameter  $d_m$ <sup>7,34</sup> influence the constants in equations 13a, 14a, and 15a. Also, the general form of equations 13a, 14a, and 15a is the same as that found in the earlier research,<sup>12</sup> except that these equations contain average cone penetration resistance  $\bar{q}_c$  instead of cone penetration resistance  $q_c$ .

44. If two cohesionless soils are assumed to have different compactibilities and constant  $a_2 = 0$ , the relation between cone penetration resistance and relative density can be plotted schematically

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\* Defined by Terzaghi:<sup>35</sup>  $D^* = (e_{\max} - e_{\min})/e_{\min}$ .

(plate 8a).<sup>2</sup> At the same relative density, cone penetration resistance increases with decreasing compactibility. From that, inclination angle  $\beta$ , whose tangent corresponds to constant  $a_1$  in equations 13a, 14a, and 15a, can be seen to increase with increasing compactibility. On the other hand, if two soils are assumed to have the same compactibility ( $\beta = \text{constant}$ ), the cone penetration resistance at the same relative density increases with the mean grain diameter (plate 8b).<sup>12</sup> Thus, the intersection on the  $D_r$  axis, which is equivalent to constant  $a_2$  in equations 13a, 14a, and 15a, decreases with increasing mean diameter.

45. These conclusions were confirmed by the results of this study. Constants  $a_1$  and  $a_2$  of equations 13-15 are summarized in the following tabulation, together with the corresponding compactibilities and mean diameters; their relations are plotted in plate 9.

Relation	Equation No.	Type of Sand	Constant $a_1$	Compactibility $D^*$ , %	Constant $a_2$	Mean Diameter $d_m$ , mm
$D_r$ versus $\bar{q}_c$	13a	Yuma	71.2	51.2	-88.6	0.12
	14a	Mortar	75.5	58.7	-106.0	0.27
	15a	Bayou Pierre	77.2	63.3	-119.2	0.50
$D_r$ versus $G$	13b	Yuma	71.1	51.2	+51.6	0.12
	14b	Mortar	75.0	58.7	+39.3	0.27
	15b	Bayou Pierre	77.0	63.3	+29.5	0.50

46. Of course, no general valid relation for the influence of compactibility and grain diameter can be derived that is based on the investigation of only three different sands. Nevertheless, a qualitative trend can be recognized, the same that was found earlier<sup>12</sup> for the resistance to penetration of three different dynamic penetrometers and one static penetrometer.

47. The same trend also was observed for the relations between cone penetration resistance gradient  $G$  and relative density  $D_r$  (plate 10). This is not surprising, since  $G$  is an average cone penetration resistance value divided by a certain depth value.

## PART V: CONCLUSIONS AND RECOMMENDATIONS

### Conclusions

48. Based on the results of this study, the following can be concluded:

- a. What occurs during the penetration of a cone into a cohesionless medium can be explained only qualitatively by theoretical means (paragraphs 9-22).
- b. The critical depth occurs in the registered 0- to 15-cm depth range (4 to 19 cm below the surface) only in loose and very loose sands (paragraph 39).
- c. Relations between relative density  $D_r$  and average cone penetration resistance  $\bar{q}_c$  or gradient  $G$  can be established by statistical means, thus facilitating communication among various agencies concerned with similar research (paragraphs 34-36).
- d. The relations between relative density and cone penetration resistance depend on the grain diameter and the compactibility of the considered cohesionless soil; so these properties have to be taken into account when the relative density of a certain soil is estimated from measurements with the cone penetrometer (paragraphs 41-47).
- e. The above-mentioned relations are valid only for the range of cone penetration measurements and soil properties investigated herein (paragraph 46).

### Recommendations

49. It is recommended that:

- a. Penetration tests be conducted on various kinds of cohesionless soils, and data from tests already performed be collected to improve the validity range for the influence of soil type on cone penetration resistance.

- b. Triaxial tests with the same soils, if available under different test conditions, e.g. with various densities and mean normal stresses, be conducted to attempt a theoretical solution of the problem and the evaluation of a method for the direct determination of the friction angle from cone penetration resistance measurements.

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Table 1  
Tests in Yuc. Sand

Test Series No.	Dry Unit Weight $\gamma_d$ $\frac{\text{kg}}{\text{m}^3}$	Void Ratio $e$	Relative Density $D_r$ %	Average Cone Penetration Resistance $\bar{q}_{c2}$ $\frac{\text{kg}}{\text{cm}^2}$	Average Penetration Resistance Gradient $G$ $\frac{\text{kg}}{\text{m}^3}$	Test Series No.	Dry Unit Weight $\gamma_d$ $\frac{\text{kg}}{\text{m}^3}$	Void Ratio $e$	Relative Density $D_r$ %	Average Cone Penetration Resistance $\bar{q}_{c2}$ $\frac{\text{kg}}{\text{cm}^2}$	Average Penetration Resistance Gradient $G$ $\frac{\text{kg}}{\text{m}^3}$
Reference 36						Reference 36 (Continued)					
4-67-001-A	15.001	0.745	55.9	70	0.7	A-68-001-1	15.001	0.731	60.5	132	1.5
2	15.204	0.724	62.7	8	0.5	8	15.252	0.710	63.4	14	3.0
3	15.477	0.696	73.0	140	1.5	9	14.710	0.732	60.6	10.0	1.1
4	15.260	0.715	64.0	145	1.6	10	15.524	0.686	74.0	100	1.0
5	15.136	0.730	60.8	128	1.3	11	14.672	0.760	52.4	100	1.2
6	15.179	0.727	61.7	140	1.5	12	14.424	0.615	84.4	76	1.0
7	15.333	0.703	67.5	156	1.6	13	15.592	0.681	70.0	100	2.2
8	15.761	0.664	82.0	271	2.3	14	14.424	0.700	64.0	7	1.1
9	15.339	0.709	67.5	136	1.5	15	14.892	0.705	61.0	4	1.1
10	15.358	0.706	68.5	125	1.3	16	15.539	0.660	74.0	10	1.5
11	14.6	0.731	60.5	132	1.5	17	15.257	0.715	64.0	100	2.0
12	15.0	0.727	61.7	144	1.4	18	15.409	0.691	63.0	242	2.2
13	0.715	65.6	153	1.6	19	15.045	0.709	67.0	100	1.0	
14	15.315	0.696	84.6	235	3.0	20	15.536	0.681	70.0	10	1.1
15	15.161	0.720	60.8	141	1.5	21	15.631	0.700	67.5	100	1.9
17	15.210	0.721	63.7	141	1.5	23	15.921	0.645	86.1	200	2.0
18	15.936	0.637	80.7	327	3.3	27	15.961	0.630	90.0	240	2.0
19	15.095	0.665	80.4	302	2.9	28	14.721	0.722	60.6	1.2	1.3
20	15.243	0.718	64.6	136	1.4	29	15.317	0.709	67.5	100	1.9
21	15.275	0.715	65.6	130	1.4	Reference 36					
22	14.140	0.730	60.8	154	1.6	11	14.177	0.645	82.5	46	0.0
23	15.16	0.727	61.7	142	1.4	5-S	14.654	0.701	60.6	82	1.0
24	15.158	0.727	61.7	150	1.0	13	14.915	0.758	51.8	90	1.1
25	15.297	0.712	66.6	146	1.5	1	14.931	0.754	53.1	117	1.3
26	15.086	0.736	58.8	145	1.5	1-S	15.135	0.730	60.2	131	1.6
27	15.212	0.721	63.7	150	1.6	2-S	15.731	0.667	81.0	310	3.6
28	15.227	0.721	63.7	142	1.5	3-S	15.936	0.645	88.1	379	4.6
29	15.402	0.701	70.1	145	1.6	7-S	16.077	0.631	92.0	441	5.6
30	15.257	0.718	64.6	151	1.5	3-S	16.124	0.620	94.2	413	5.0
31	15.125	0.733	59.8	150	1.5	4-S	16.171	0.621	95.6	470	5.8
33	16.248	0.613	98.4	416	4.5	Control Tests					
34	15.857	0.653	85.5	359	4.2	Y1	14.254	0.636	80.0	26	0.3
35	16.034	0.634	91.6	433	4.7	Y2	14.676	0.780	42.8	1	0.7
37	16.023	0.634	91.6	412	4.4	Y3	14.150	0.652	81.0	33	0.4
38	15.946	0.642	89.1	419	4.5	Y4	14.018	0.694	66.1	35	0.4
39	16.049	0.613	93.4	438	4.7	Y5	14.234	0.642	84.6	29	0.4
42	16.279	0.610	99.4	473	5.1	Y6	13.627	0.824	60.0	26	0.3
43	15.978	0.633	90.0	282	3.0	Y7	13.773	0.901	50.6	28	0.3
44	15.837	0.656	84.0	245	2.4	Y8	14.440	0.616	85.4	72	0.0
45	16.234	0.615	97.7	321	3.5	Y9	14.440	0.615	85.4	49	0.5
46	15.899	0.648	87.1	277	2.9	Y10	13.632	0.844	60.0	2	0.4
47	15.977	0.642	89.1	244	2.0	Y11	14.542	0.607	90.7	112	1.2
A-68-001-1	15.970	0.639	90.0	236	2.0	Y12	15.226	0.721	67.0	100	1.7
2	16.110	0.620	94.2	243	2.5	Y13	15.453	0.702	70.0	100	2.1
3	16.045	0.634	91.0	249	2.0	Y14	15.450	0.700	70.0	210	2.3
4	16.127	0.623	95.2	235	3.0	Y15	15.902	0.640	87.0	201	3.1
5	16.237	0.613	96.4	333	3.0						

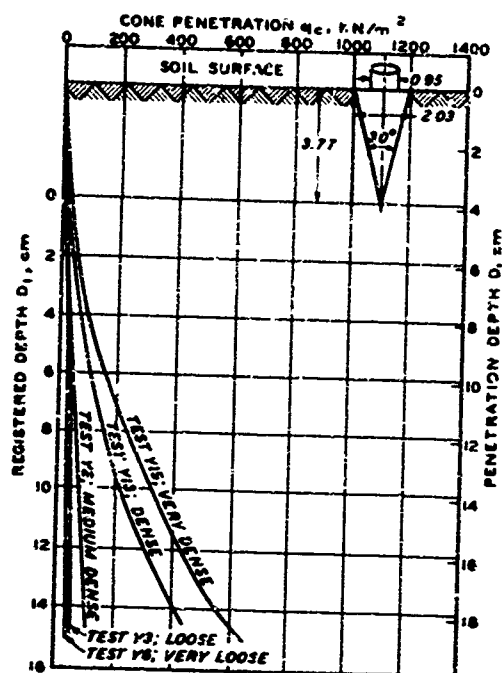
Table 2  
Tests in Mortar Sand

Test Series No.	Dry Unit Weight $\gamma_d$ , kN/m <sup>3</sup>	Void Ratio $e$	Relative Density $D_r$ , %	Average Cone Penetration Resistance $\bar{q}_c$ , kN/m <sup>2</sup>	Average Penetration Resistance Gradient $G$ , MN/m <sup>3</sup>
<u>Reference 16</u>					
548	14.852	0.764	42.9	76	0.8
5	14.805	0.770	41.1	90	1.0
12	14.931	0.754	45.8	83	0.9
--*	14.962	0.751	46.7	131	1.7
549	15.041	0.742	49.4	96	1.1
--*	15.370	0.704	60.7	117	1.5
550	15.370	0.704	60.7	158	1.6
551	15.355	0.706	60.1	179	2.0
--*	15.386	0.704	60.7	207	2.7
7	15.402	0.701	61.6	207	2.2
552	15.449	0.695	63.5	193	2.1
554	15.794	0.658	74.4	227	2.7
--*	15.810	0.658	74.4	193	2.5
553	15.888	0.650	76.8	241	2.5
--*	15.983	0.639	80.1	345	4.5
10	16.030	0.634	81.5	310	3.7
9	16.108	0.626	83.9	338	4.0
6	16.187	0.618	86.3	303	3.2
--*	16.202	0.618	86.3	296	3.9
556	16.281	0.610	88.7	282	3.3
555	16.312	0.608	89.3	262	3.0
557	16.328	0.605	90.1	358	4.1
558	16.422	0.595	93.2	372	4.3
<u>Control Tests</u>					
M1	14.117	0.855	15.8	39	0.5
M2	14.117	0.855	15.8	38	0.4
M3	13.881	0.887	6.3	35	0.4
M4	15.206	0.721	55.7	130	1.4
M5	14.754	0.776	39.3	88	1.0
M6	14.421	0.815	27.7	68	0.7
M7	13.949	0.876	9.5	43	0.6
M8	16.039	0.634	81.5	350	4.7
M9	15.706	0.667	71.7	269	3.7
M10	15.892	0.647	77.7	330	4.6
M11	16.206	0.616	86.9	438	6.3
M12	16.412	0.595	93.2	419	5.8
M13	16.353	0.603	90.8	455	6.3
M14	16.402	0.597	92.6	435	6.0

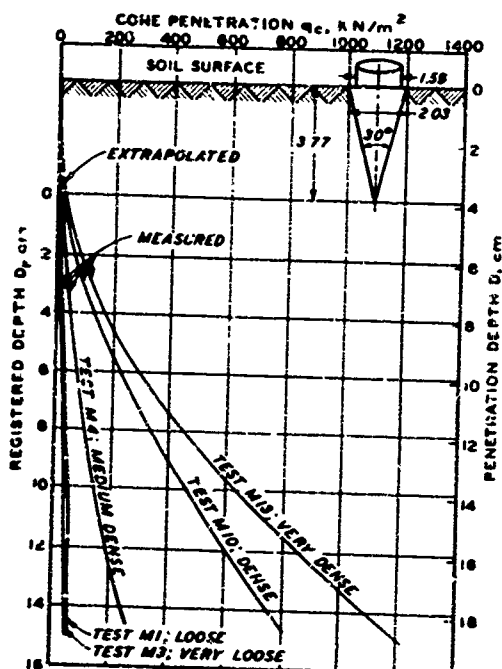
\* Tests were not numbered.

Table 3  
Tests in Bayou Pierre Sand

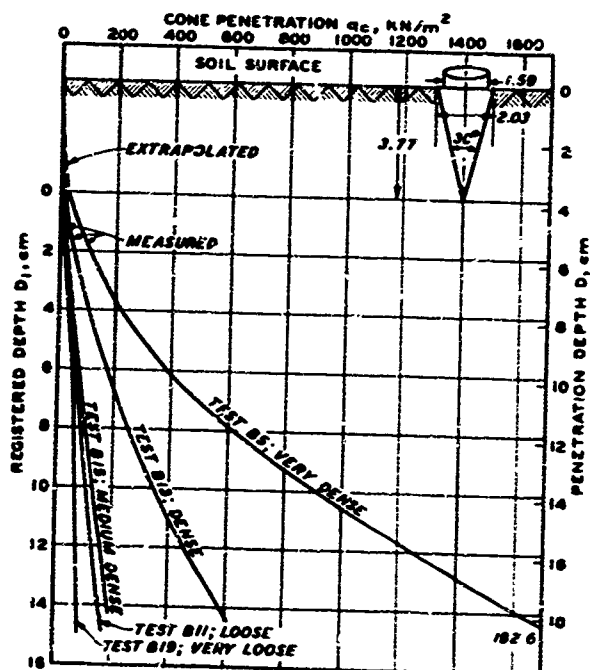
Test Series No.	Dry Unit Weight $\gamma_d$ , kN/m <sup>3</sup>	Void Ratio e	Relative Density $D_r$ , %	Average Cone Penetration Resistance $\bar{q}_c$ , kN/m <sup>2</sup>	Average Pene- tration Re- sistance Gradient G, kN/m <sup>3</sup>
B1	15.676	0.658	0.0	59	0.7
B2	16.245	0.600	22.8	79	0.9
B3	16.294	0.595	24.7	79	0.9
B4	17.874	0.439	85.9	406	4.1
B5	18.423	0.410	97.3	598	9.1
B6	18.060	0.439	85.9	406	5.2
B7	18.237	0.425	91.4	703	9.2
B8	18.266	0.422	92.5	681	8.6
B9	17.717	0.466	75.3	342	4.3
B10	16.069	0.618	15.7	57	0.6
B11	16.196	0.605	20.8	76	0.9
B12	17.305	0.502	61.2	176	2.2
B13	17.609	0.477	71.0	261	3.4
B14	15.980	0.626	12.5	59	0.7
B15	16.775	0.550	42.4	102	1.2
B16	17.118	0.520	54.1	158	1.9
B17	16.039	0.621	14.5	55	0.6
B18	16.893	0.538	47.1	113	1.2
B19	15.971	0.629	11.4	53	0.6
B20	16.942	0.537	47.5	129	1.5
B21	16.961	0.534	48.6	128	1.5
B22	17.942	0.449	82.0	405	5.1
B23	17.982	0.445	83.5	405	5.0
B24	16.579	0.567	35.7	108	1.2
B25	16.393	0.585	28.6	74	0.9
B26	16.569	0.570	34.5	102	1.2
B27	17.374	0.497	63.1	205	2.4
B28	17.609	0.477	71.0	242	3.0
B29	17.462	0.488	66.7	205	2.6
B30	17.423	0.493	64.7	185	2.3
B31	18.060	0.439	85.9	419	4.9
B32	18.129	0.435	87.5	513	6.4
B33	18.276	0.422	92.5	526	6.5
B34	18.188	0.429	89.8	457	5.8
B35	16.520	0.575	32.5	85	1.0
B36	17.727	0.466	75.3	269	3.4



a. YUMA SAND



b. MORTAR SAND



c. BAYOU PIERRE SAND

REPRESENTATIVE TEST  
RESULTS IN  
THREE TEST SANDS

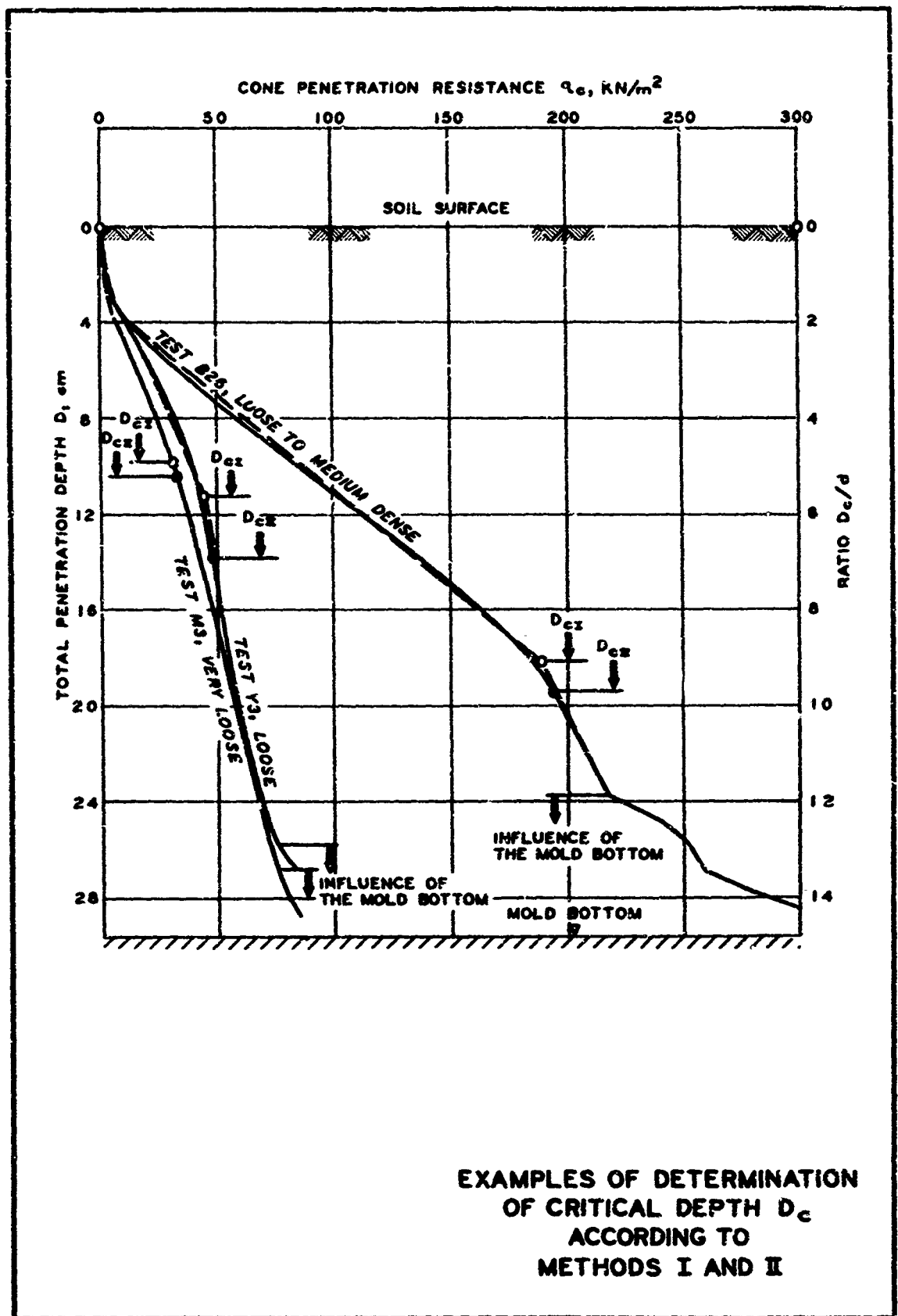
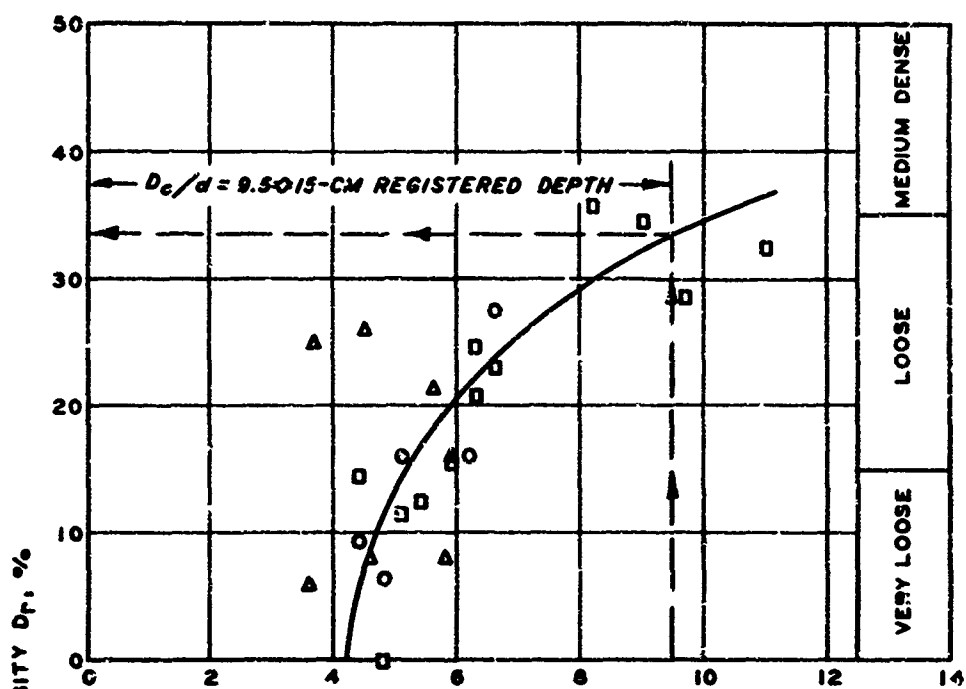
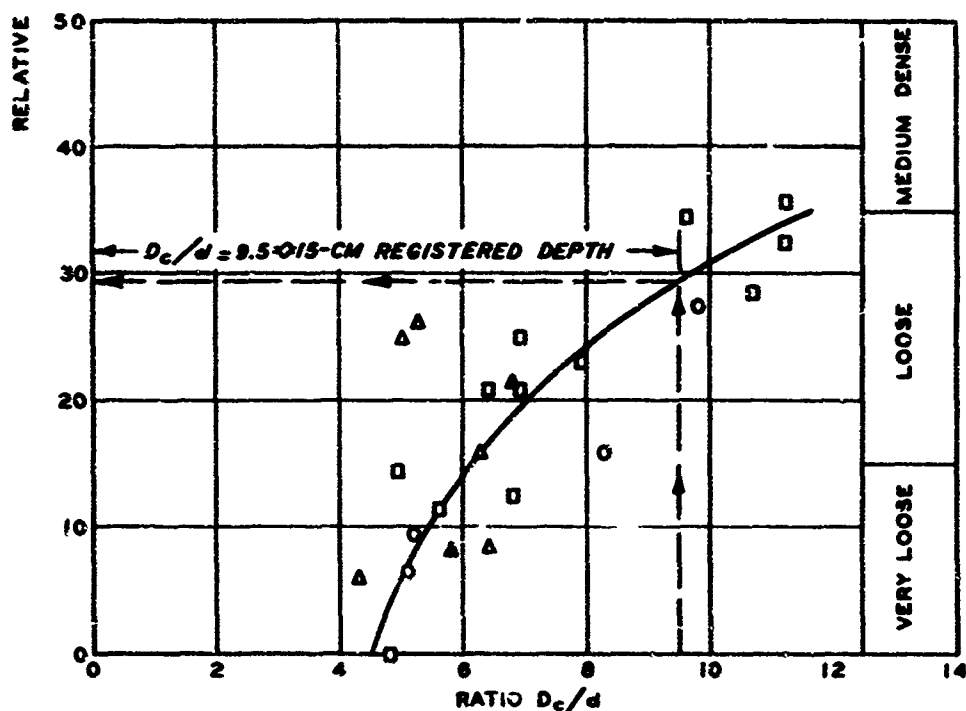


PLATE 2



a. ACCORDING TO METHOD I

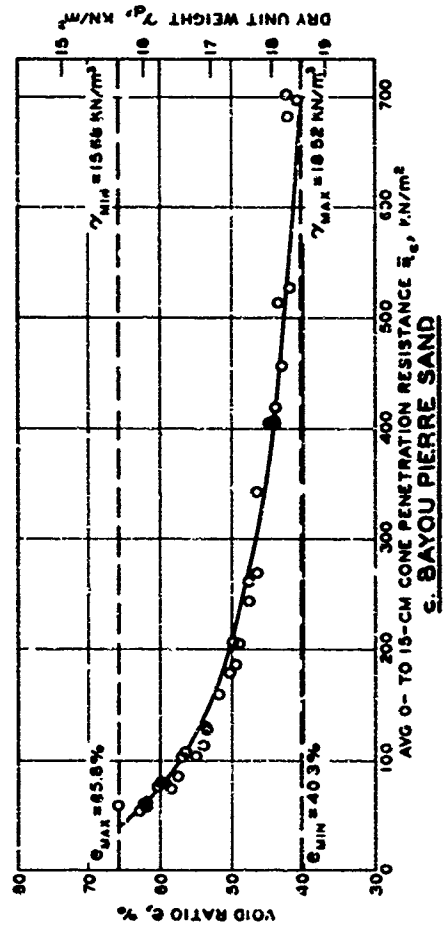
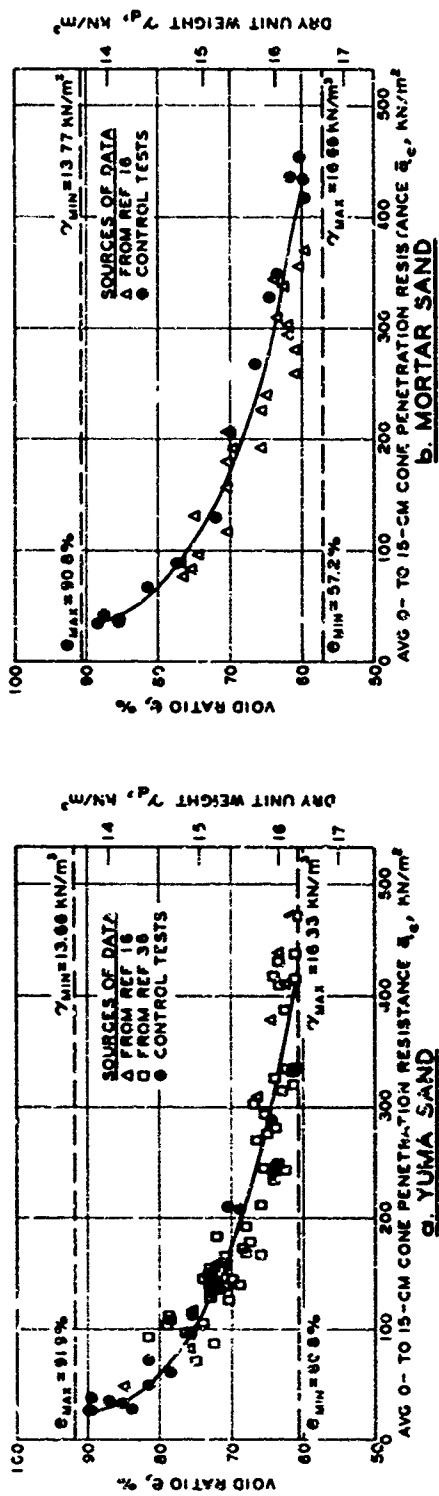


b. ACCORDING TO METHOD II

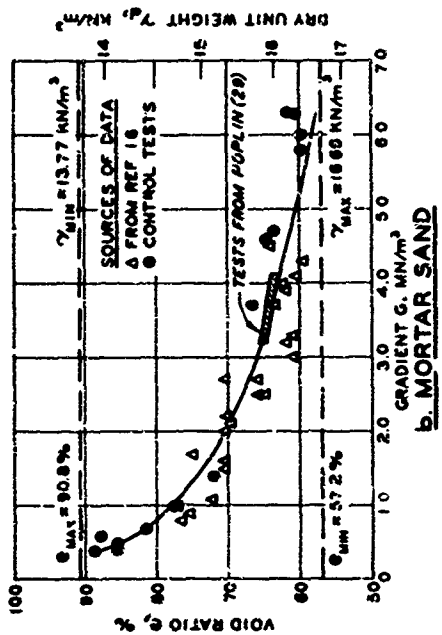
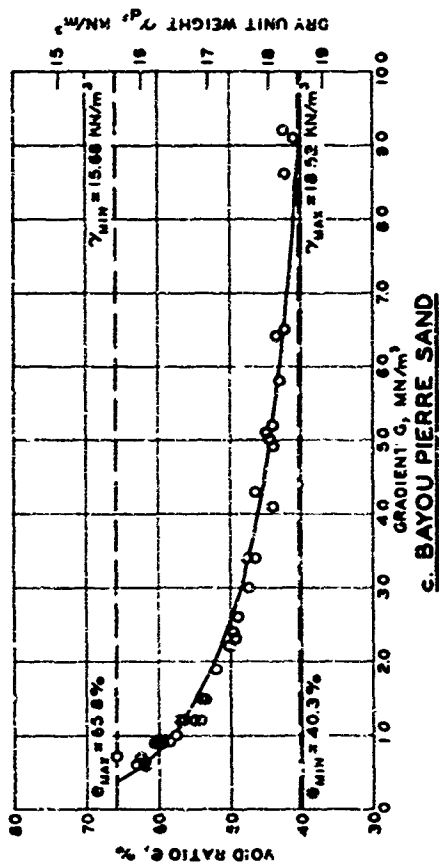
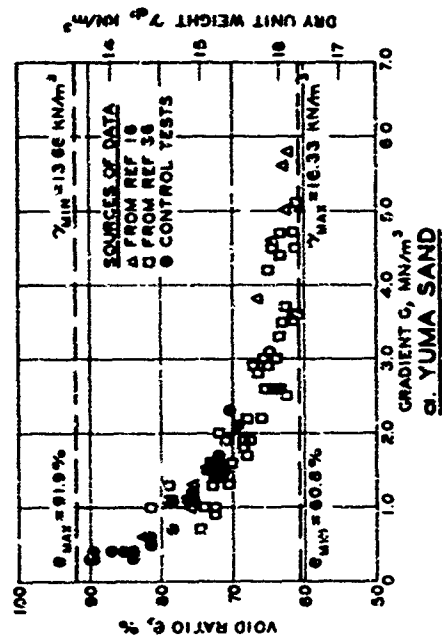
**LEGEND**

- △ YUMA SAND
- MORTAR SAND
- BAYOU PIERRE SAND

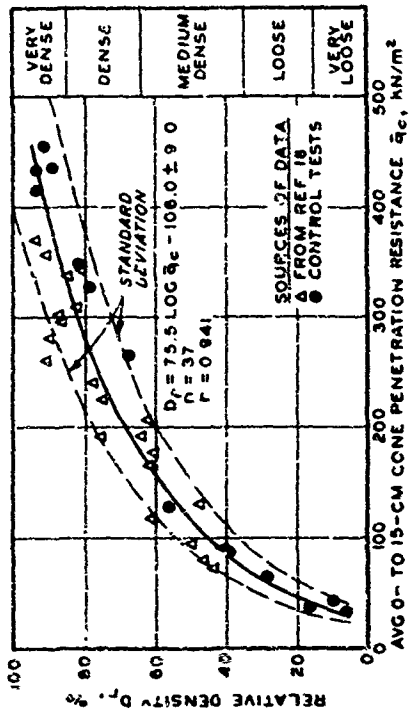
RELATION BETWEEN  
RELATIVE DENSITY  
 $D_r$  AND RATIO  $D_c/d$



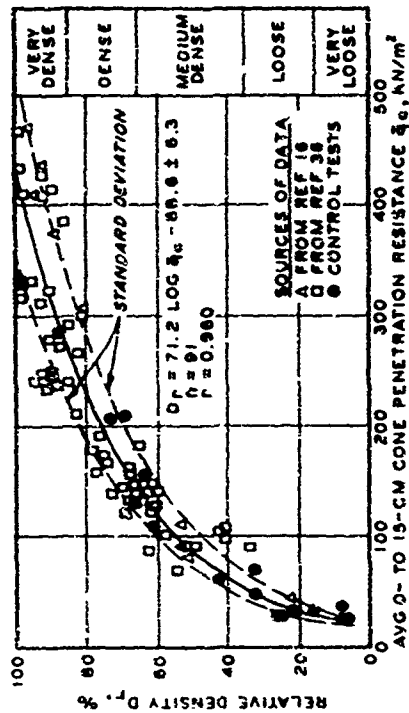
RELATIONS  
AMONG VOID RATIO,  
DRY UNIT WEIGHT, AND AVERAGE  
CONE PENETRATION RESISTANCE



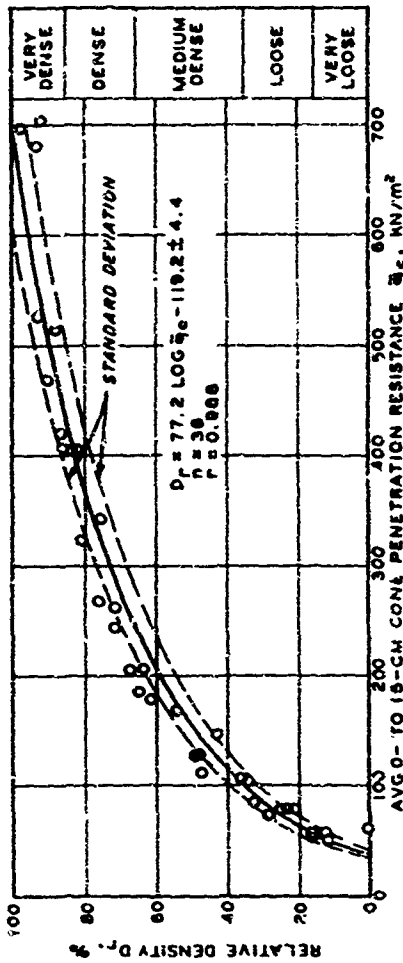
RELATIONS  
 AMONG VOID RATIO,  
 DRY UNIT WEIGHT, AND  
 PENETRATION RESISTANCE GRADIENT



**b. MORTAR SAND**

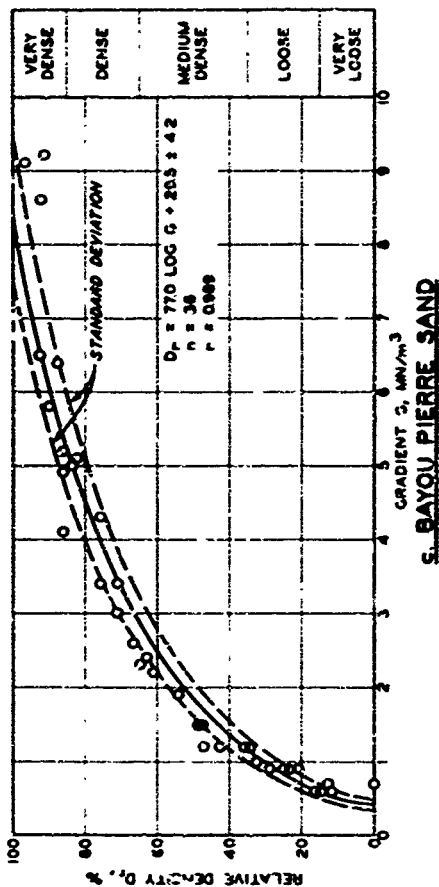
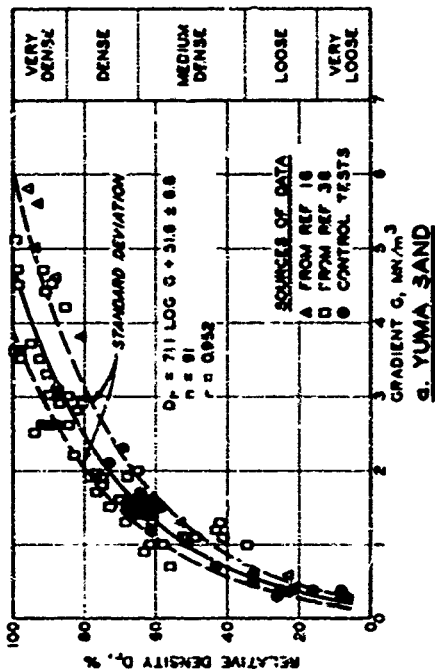
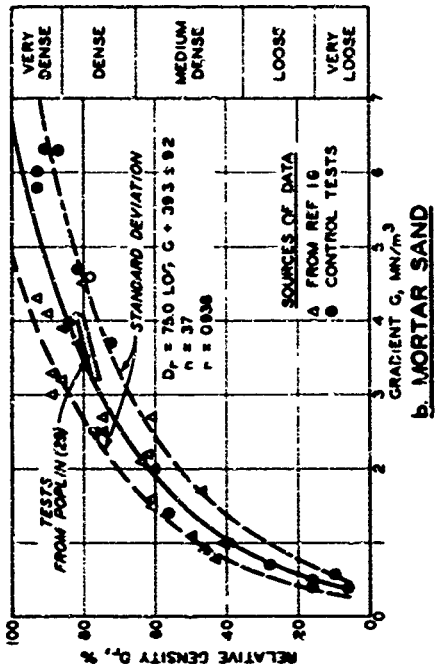


**a. YUMA SAND**

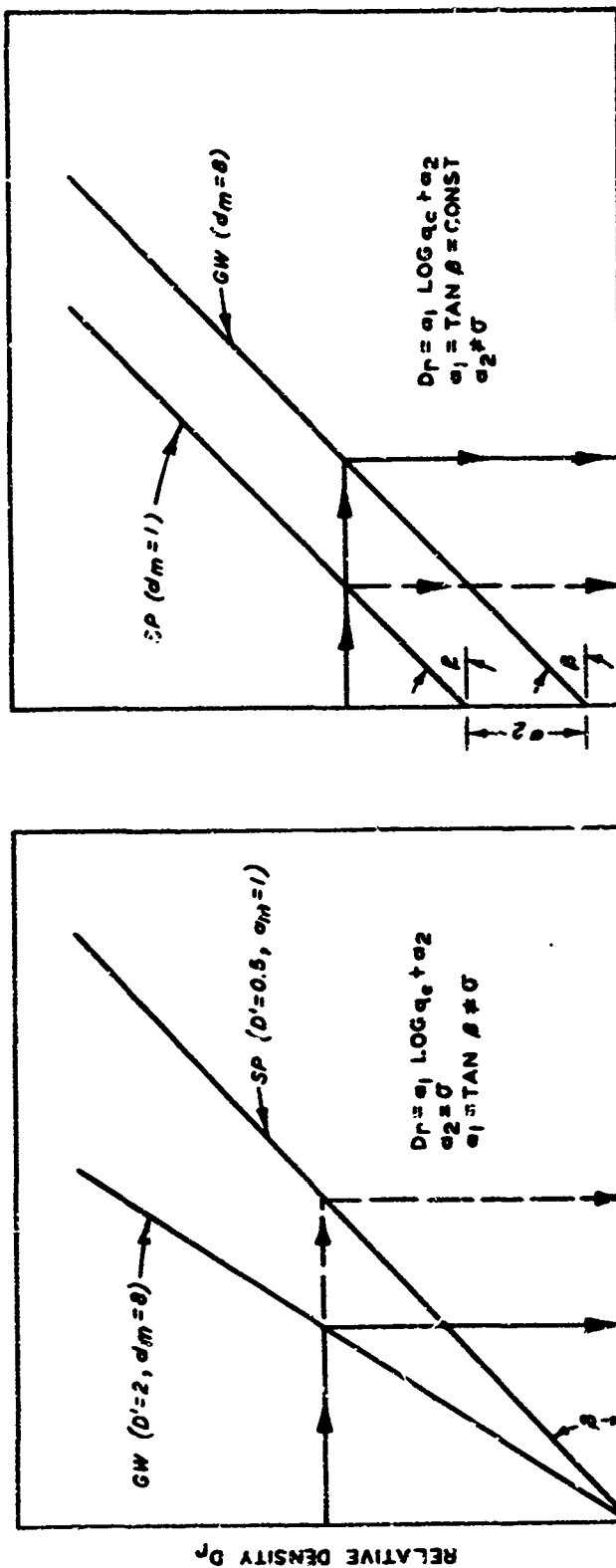


**c. BAYOU PIERRE SAND**

**RELATION  
BETWEEN RELATIVE  
DENSITY AND AVERAGE  
CONE PENETRATION RESISTANCE**



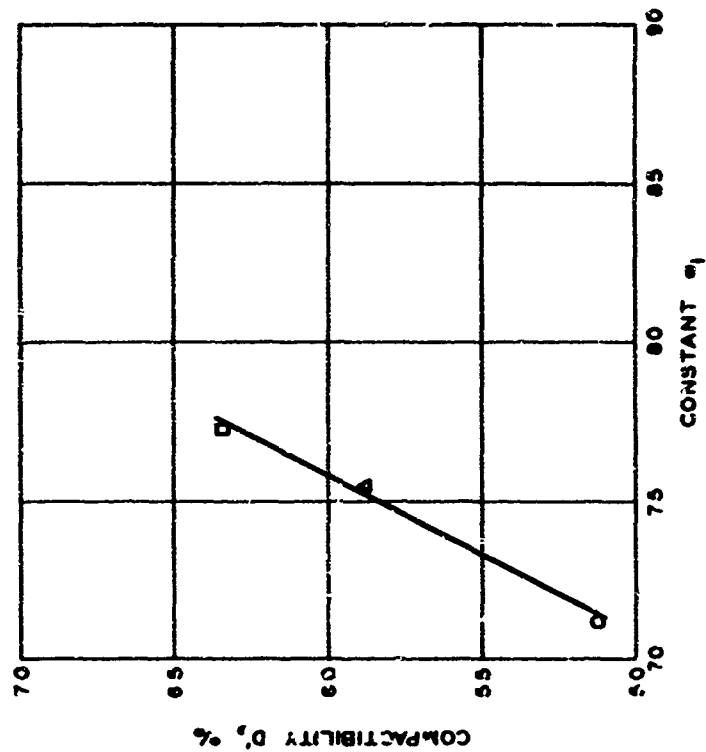
# RELATION BETWEEN RELATIVE DENSITY AND PENETRATION RESISTANCE GRADIENT



a. RELATION BETWEEN  $\sigma_1$  AND  $D'$  b. RELATION BETWEEN  $\sigma_2$  AND  $d_m$

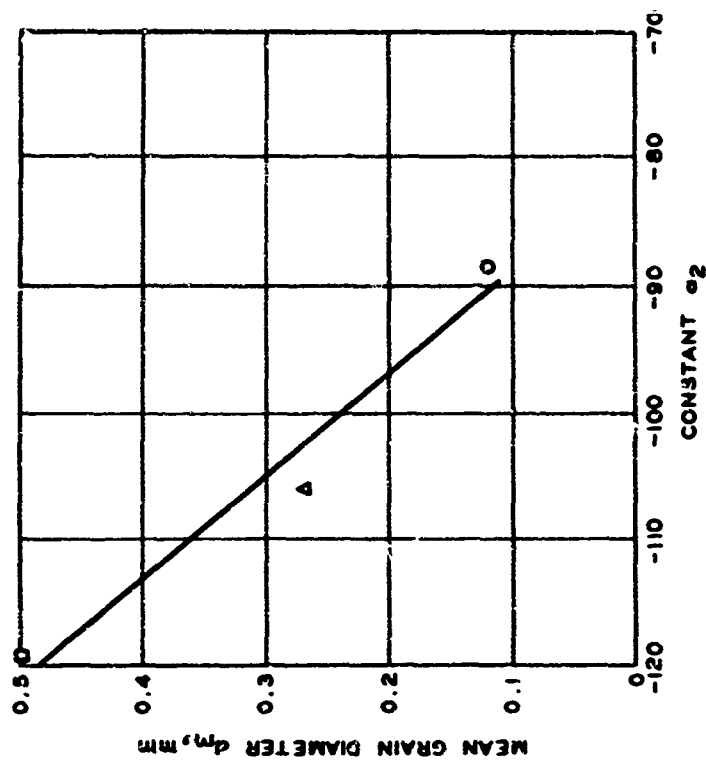
NOTE: SCHEMATIC FROM REF 12.

INFLUENCE OF COMPACTIBILITY  
AND GRAIN DIAMETER ON  
CONE PENETRATION RESISTANCE



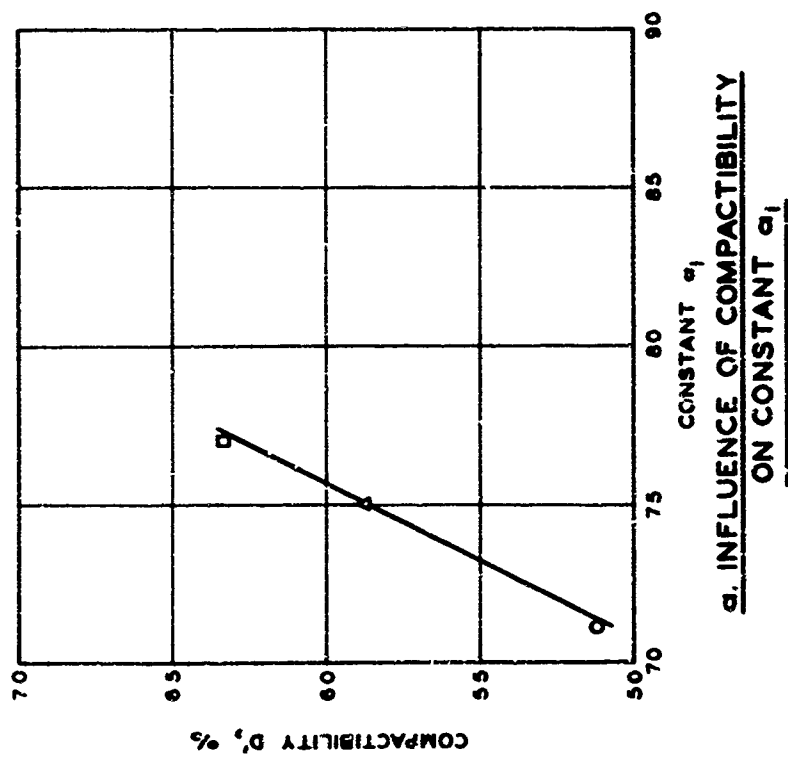
a. INFLUENCE OF COMPACTIBILITY  
ON CONSTANT  $\sigma_1$

**LEGEND**  
O YUMA SAND  
A MORTAR SAND  
n BAYOU PIERRE SAND



b. INFLUENCE OF MEAN GRAIN DIAMETER  
ON CONSTANT  $\sigma_2$

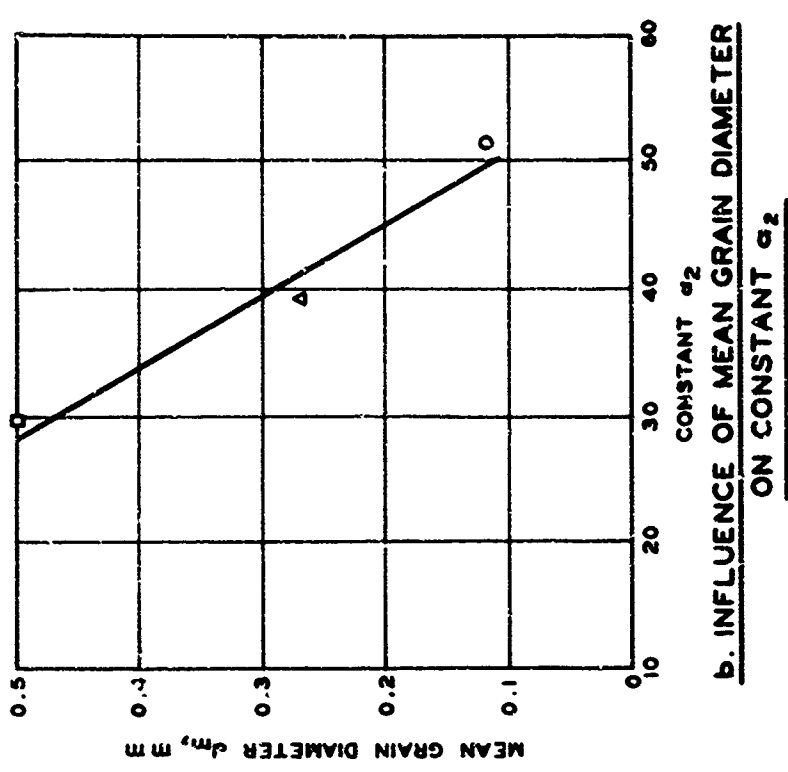
**FACTORS AFFECTING RELATION  
BETWEEN RELATIVE DENSITY  
AND AVERAGE CONE  
PENETRATION RESISTANCE**



a. INFLUENCE OF COMPACTIBILITY  
ON CONSTANT  $a_1$

LEGEND

- O YUMA SAND
- Δ MORTAR SAND
- BAYOU PIERRE SAND



b. INFLUENCE OF MEAN GRAIN DIAMETER  
ON CONSTANT  $a_2$

FACTORS AFFECTING RELATION  
BETWEEN RELATIVE DENSITY  
AND PENETRATION  
RESISTANCE GRADIENT